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THE DYNAMIC RESPONSE OF WIND TURBINE GENERATORS FLEXIBLE STRUCTURES TO THE TURBULENT WIND EFFECT

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ABSTRACT

The high risk of construction-assembly activities, the high costs of leased equipment, the pressure to finish installing/uninstalling as quickly as possible, create an environment conducive to ignoring safety practices, especially if the rules do not become mandatory, even if they are known.

The purpose of this article is to analyze the response of a wind turbine metallic tubular tower to wind action when, for various reasons, during operation, dismantling or installation, circumstances can arise that leave the structure to remain in a preserved state in different stages.

The dynamic response factor of the structure was evaluated using the Romanian National Design Code and Eurocode 1. The relationships are based on the hypothesis that only the vibrations of the structure in the direction of the wind, corresponding to the fundamental mode of vibration, are significant.

Because the structure presented is isolated, high and slender, the article also examines the probability of occurrence of interaction effects between vortex shedding and galloping.

Keywords: wind energy, extreme values, wind Constanta County, wind turbine generators

1. INTRODUCTION

It is known that "Wind Industry" is still a young industry that lends experience and knowledge from more developed areas. Industry practices must be carried out under national safety and quality standards and must be ensured through the quality system. This control is difficult to achieve in the absence of a framework limiting clear and transparent rules to possible situations that may affect both the structural safety and the safety of employees working in temporary mobile sites.

The high degree of risk of construction-assembly activities (working at heights, in narrow spaces, high loads at high heights, the possibility of impact, physical exhaustion etc.), the high cost of equipment, rented equipment, the pressure to finish as long as possible installing/removing quickly, creates a favorable environment where safety issues can be easily ignored, especially if are not mandatory, even if are known.

In many cases safety criteria and execution procedures are adapted after the incidents are in place, but we can anticipate events and increase safety by setting rules not only based on accident statistics.

The aim of this paper is to analyze the wind turbine generators tower's response to wind action when, for various reasons during operation, dismantling or installation, circumstances can arise that will keep the structure for a long period into a preservation regime in various intermediate stages of realization. Some of the most foreseeable reasons are: failure of the generator that leads to change or ground repair, delays in delivery of the components during assembly, arrival in site of upper elements showing nonconformities, owner's decision to move the generator or to change the generator.

The motivation comes from the fact that there are over 1500 wind turbines with metal tubular towers installed in Romania. Considering that the lifetime of a generator is 20 years, the desire to refurbish, decommission, relocate, change the generator to existing projects will appear next to new installations. These strategies will be taken by each operator independently and, in the absence of technical constraints, solutions will have a profit-maximizing character without anticipating safety issues.

The metallic wind turbine tower is to be located in Constanta County (in terrain category class I according with [3], [6]) and consists of five sections of different lengths and has a total height of 102.92 m. The diameter at the base of the tower is 4.472 m and 2.316 m. The assembling of the sections is made thru flanges, with screws. The tower installation starts with the first section being mounted on the foundation, and the following sections are installed successively in following stages. At the top of the tower is mounted the wind generator composed of rotor and nacelle. Disassembly is done in reverse installation steps.

Table 1. Diameters, weights, lengths of tower sections

Tower section	Lengths [m]	Diameter at the bottom [m]	Diameter at the top [m]	Cumulated weight [kN]
Section 1	15.16	4.472	3.972	637.43
Section 2	17.64	3.972	3.634	1245.44
Section 3	20.44	3.634	3.280	1860.00
Section 4	23.52	3.280	2.883	2324.17
Section 5	26.16	2.883	2.316	2750.76

2. THEORETICAL FRAMEWORK

The dynamic response factor of a structure, accounts the amplification of wind effects due to quasi-resonant structure vibrations with the frequency content of the atmospheric turbulence combined with the reduction of the effects of wind action due to the non-simultaneous occurrence of peak wind pressures over the surface of the structure or element. The amplification of the structural response is even greater as the structure is more flexible, lighter and with a small damping. The reduction of the structural response due to the non-simultaneous occurrence of peak wind pressure values is even more pronounced as the surface of the building exposed to the wind action is higher.

Based on the hypothesis that only the vibrations of the structure in the direction of the wind, corresponding to the fundamental vibration mode, are significant, the procedure for the detailed evaluation of the dynamic response coefficient of the structure according with the Romanian National Design Code and the Eurocode CR 1-1-4/2012 uses the relationship:

$$C_d = \frac{1+2k_p I_v(z_s)\sqrt{B^2+R^2}}{1+7I_v(z_s)} \qquad (1)$$

where

z_s is the reference height for determining the dynamic response; for cases not shown in Codes, z_s can be taken as equal to the height of the structure,

k_p is the peak factor for computing the peak response of the structure with respect to the mean response averaged on $t = 10\ min = 600\ s$,

B^2 is the background response part, which evaluates the correlation of wind pressures on the surface of the building,

R^2 is the resonant response part, which evaluates the dynamic amplification effects of the structural response produced by the frequency content of the turbulence in quasi-resonance with the fundamental vibration frequency of the structure,

$I_v(z_s)$ is the intensity of turbulence at height $z = z_s$.

Dynamic response factors above unit, $C_d > 1$, occurs when the equivalent static action is greater than the peak aerodynamic action. This situation can materialize also in the case of a significant resonance response, when the dynamic response produced by the resonant oscillations of the structure can bring displacements and stresses with greater values as the structure is more flexible and with a lower damping.

The dynamic response factor applies to resulting forces and external pressures in the direction of the wind (along-wind).

For slender, tall and isolated structures, it is also necessary to consider the dynamic effect produced by vortex shedding. This phenomenon produces a fluctuating action perpendicular to the direction of the wind with a frequency depending on the average wind speed and the shape and dimensions of the cross-section of the structure.

If the vortex frequency is close to the structure frequency, the quasi-resonance conditions that produce increasing amplitudes to the structure can be satisfied, especialy if the damping and the mass of the structure or the element presents small values.

The effect of vortex shedding will be considered during the design phase according to [3], [6] if the below condition applies:

$$v_{crit,i} < 1.25 v_m \qquad (2)$$

where $v_{crit,i}$ is the critical wind velocity for vibration mode i and v_m is the characteristic mean wind velocity at the cross section where vortex shedding occurs.

The critical wind velocity can be estimated using the formula:

$$v_{crit,i} = \frac{b \cdot n_{i,y}}{S_t}, \qquad (3)$$

where:

b is the reference width of the cross-section at which resonant vortex shedding occurs and where the modal deflection is maximum for the structure or structural part considered; for circular cylinders the reference width is the outer diameter;

$n_{i,y}$ is the natural frequency of the considered flexural mode i of cross-wind vibration,

S_t is the Strouhal number.

One of the most important and at the same time with a high degree of uncertainty about the dynamic response of tall structures is the structural damping. There is no theoretical method for damping estimations, the values measured on real scale models showing a dispersion of data, dispersion caused by different causes: soil type, foundation type, materials used, inappropriate measurement techniques etc. [12÷16].

In [3], for the fundamental bending mode, the logarithmic decrement of the damping, D_t, is estimated with the relation:

$$D_t = d_s + d_a + d_d \tag{4}$$

where:

d_s is the logarithmic decrement of structural damping,

d_a is the logarithmic decrement of aerodynamic damping for the fundamental mode;

d_d is the logarithmic decrement of damping due to special devices;

The characteristic peak accelerations, a_{max}, are obtained by multiplying the standard deviation, σ, by the peak factor using the natural frequency, n_{1x}. The characteristic peak acceleration values are given by the relation:

$$a_{max} = \left[\sqrt{2\ln\left(600 \cdot n_{1x}\right) + \frac{0.5772}{2\ln\left(600 \cdot n_{1x}\right)}}\right] \cdot \sigma \tag{5}$$

The standard deviation, σ, of the characteristic along-wind acceleration at height z can be obtained using:

$$\sigma = \frac{c_f \cdot \rho_{air} \cdot b \cdot I_v \cdot v_m^{MR/10^2}}{M} \sqrt{R^2} K_x \Phi_z \tag{6}$$

where

C_f is the force coefficient,

b is the width of the structure,

I_v is the turbulence intensity,

$v_m^{MR/10}$ is the mean wind velocity with a 10 year mean recurrence interval,

R^2 is the resonant response,

K_x is the non-dimensional coefficient, according with Codes,

M is the along wind fundamental equivalent mass,

ϕ_z is the fundamental along wind modal shape.

To ensure optimum installation conditions, the constrain shown in the below formula was verified:

$$a_{max} < a_{lim} \tag{7}$$

where a_{max} is defined above,

a_{lim} is the upper limit of comfort.

The expected frequency of gust loading on structures [11] can be estimated using the expression:

$$f_0\left[Hz\right] = \frac{v_{mz}}{L_t\left(\dfrac{z_s}{z_t}\right)^{\alpha}} \frac{1}{1.11 \cdot S^{0.615}} \tag{8}$$

where S is the factor for computing the expected frequency of gust given by:

$$S = 0.46\left(\frac{b+h}{L_t\left(\dfrac{z_s}{z_t}\right)^{\alpha}}\right) + 10.58\frac{\sqrt{b \cdot h}}{L_t\left(\dfrac{z_s}{z_t}\right)^{\alpha}} \tag{9}$$

The sensibility to the vibrations is interconnected with the structural damping and the ratio of structural mass to the air mass. The Scruton number, S_c, an dimensionless parameter that depends on the equivalent mass, damping and the dimension of the section is given by:

$$S_c = \frac{2M \cdot d_s}{\rho_{air} \cdot b^2} \tag{10}$$

where M is the equivalent mass, d_s is the structural damping expressed by the logarithmic decrement, ρ_{air} is the air density under vortex shedding conditions, recommended 1.25 kg/m^3, b is the reference width of the cross-section at which resonant vortex shedding occurs.

According with [3], [6] when the critical wind velocity estimated with (3) has a value close to the onset wind velocity of galloping, interaction effects between vortex shedding and galloping are likely to occur. This condition is underlined by the expression:

$$0.7\left\langle < \frac{v_{CG}}{v_{crit,i}} < 1.5 \right\rangle \tag{11}$$

where v_{CG} is the onset wind velocity of galloping, given by the formula:

$$v_{CG} = \frac{2S_C}{a_G} n_{1,y} \cdot b \tag{12}$$

where S_C is the Scruton number, $n_{1,y}$ is the cross-wind fundamental frequency of the structure, b is the width, a_G is the factor of galloping instability.

1. CASE STUDY RESULTS

By paying more attention to the achievement of safety during the assembling activities that take place under the stochastic actions of the wind, after analyzing the installation/dismantling stages for the wind turbine generators tower sections designated to be placed in Constanta County, with the characteristics presented in Table 1. The peak acceleration values at the tip of each section are shown in Table 3.

The values shown in Table 2 were obtained for C_d. Results meeting the comfort criteria are obtained at the wind speeds presented under Table 4, where the wind speed is averaged over 10 minutes at 10 meters height.

The expected frequency of gust loading, the critical wind velocity, the Scruton number and the onset wind velocity of galloping are shown in Tables 5÷7.

Table 2. The dynamic response factor C_d

Installed sections	Height h [m]	Fundamental frequency of the structure along-wind n_{1x} [Hz]	The dynamic response factor C_d [-]
5 sections	102.92	0.208	1.10
4 sections	76.76	0.468	1.13
3 sections	53.24	1.109	1.12
2 sections	32.80	3.237	1.03

Table 3. Along-wind acceleration

Installed sections	Height h [m]	a_{max} [m/s^2]
5 sections	102.92	1.22
4 sections	76.76	1.13
3 sections	53.24	0.89
2 sections	32.80	0.49

Table 4. Along-wind acceleration under the installation conditions

Installed sections	**Height h [m]**	**v_b [m/s]**	$a_{max\ reduced} < a_{max}$ **[m/s^2]**
5 sections	102.92	11	0.13
4 sections	76.76	10	0.08
3 sections	53.24	10.5	0.06
2 sections	32.80	16.5	0.09

Table 5. The expected frequency of gust loading

Installed sections	**Height h [m]**	**Expected frequency of gust loading, f_0, [sec]**	**Expected frequency of gust loading f_0, [Hz]**	**Factor S [-]**
5 sections	102.92	5.29	0.189	1.18
4 sections	76.76	4.93	0.203	1.23
3 sections	53.24	4.41	0.227	1.23
2 sections	32.80	3.75	0.267	1.22

Table 6. The Scruton number

Installed sections	**Height h [m]**	**S_c [-]**
5 sections	102.92	5.12
4 sections	76.76	4.12
3 sections	53.24	3.62
2 sections	32.80	3.22

Table 7. The onset wind velocity of galloping

Installed sections	**Height h [m]**	**Onset wind velocity of galloping v_{CG} [m/s]**	$\frac{v_{CG}}{v_{crit,i}}$ **[-]**
5 sections	102.92	4.95	1.84
4 sections	76.76	11.11	1.48
3 sections	53.24	26.39	1.30
2 sections	32.80	The phenomenon no longer needs investigation	

2. CONCLUSIONS

The dynamic response factor of the structure is greater than 1 in all the analyzed steps, indicating a poor damping of the structure. The values of the dynamic response coefficient of the structure do not increase with the height of the tower (Table 7).

Without additional energy dissipation devices, the total logarithmic decrement of the tower is 0.178 and corresponds to an approximate 2.8% total damping coefficient value, which is within the range obtained from measurements on tall buildings 0.6% ÷ 3.4%. In order to reduce oscillations and optimize damping, energy dissipation elements (liquid dampers etc.) can be introduced. A 20% increase of the total decrement would reduce the total structural damping coefficient to 3.4%.

Note that the roughness of the tower wall may influence the value of the dynamic response factor. Being aware that the fundamental frequency of the structure along-wind, n_{1x}, has a value close to the expected frequency of gust loading, f_0, for the structure with 5 sections, as shown in the Table 8, and considering that in the case of a difference of less than 33% between the expected frequency of gust loading and the fundamental frequency of the structure, the structural dynamic response to those gusts that have a duration close to the fundamental period of the structure becomes an amplification factor which can lead to resonance, phenomenon occurring in any flexible structure under the influence of wind, the paper proposes, in Table 9, the level of safety to be adopted in the installation/dismantling of the towers for the wind turbines considering only the dynamic action of the wind, where the main parameter has a variable governed by probability calculation.

Table 8. Fundamental frequency of structure along-wind, and the expected frequency of gust loading

Installed sections	Height h [m]	The expected frequency of gust loading f_0 [Hz]	The fundamental frequency of structure along-wind n_{1x} [Hz]
5 sections	102.92	**0.189**	**0.208**
4 sections	76.76	0.203	0.468
3 sections	53.24	0.227	1.109
2 sections	32.80	0.267	3.237

Table 9. Wind speed limit proposed for installation activities

Installed sections	**Height h [m]**	**Wind speed limit for installation activities [m/s] 1) 2)**	**f_0 [Hz]**	**Fundamental frequency of the structure along-wind n_{1x} [Hz]**
5 sections	102.92	**20 m/s**	**0.134**	**0.208**
4 sections	76.76	-	0.203	0.468
3 sections	53.24	-	0.227	1.109
2 sections	32.80	-	0.267	3.237

1) averaged over a 10-minute period, determined at a height of 10 m and having annual probabilities of exceedance of 0.02.

2) for regions with peak velocity pressure $q_b=0.5$ according with CR 1-1-4/2012.

f_0 - maximum expected frequency of gust loading

Particular attention should be paid to work interruptions after installing the sections. For whatever reason (technical, financial, legal), the interruption of activities should not last for more than a few hours. If this interval can not be met, the last section must be dismantled and secured at ground level. At the same time, it is observed that the wind gust frequency tends to have higher values at lower altitudes.

The characteristic peak accelerations were obtained by multiplying the standard deviation with the peak factor calculated with the fundamental vibration frequency of the structure in the wind direction. The effects of the wind on the structure must not cause oscillations with amplitudes and frequencies that create dangerous situations.

Considering that optimal installation conditions can be achieved by fulfilling the condition of formula (10) and given that peak acceleration values above 0.2 m/s^2 can cause balance loss to people, the paper proposes, in Table 10, the maximum wind speeds (wind speed averaged over 10 minutes at 10 meters height) at which to install the tower, safely.

Table 10. Maximum wind speeds for installation activities

Installed sections	Height h [m]	v_b [m/s]	$a_{max\ reduced} < a_{max}$ [m/s^2]
5 sections	102.92	**11**	0.13
4 sections	76.76	**10**	0.08
3 sections	53.24	**10.5**	0.06
2 sections	32.80	16.5	0.09

High values of the peak characteristic acceleration are present at heights of 32.80 m, reason why the paper does not recommend, for any reason (technical, financial, legal), interruption of work at heights above 32.80 m for a period longer than 3 days. If this interval can not be met, all elements over 32.80 m have to be dismantled and secured at ground level. It is not advisable to keep the elements up to 32.80 m for more than a year, after which, if the work is not resumed, it is recommended to be dismantled and secured on the ground.

Table 11. Peak characteristic acceleration for each section

Installed sections	**Height h [m]**	a_{max} **[m/s^2]**
5 sections	102.92	1.221
4 sections	76.76	1.131
3 sections	53.24	0.892
2 sections	**32.80**	**0.493**

The mall values of the Scruton number ($S_c<5.2$) indicates that the vibrations induced by the vortex shedding can be of great amplitude.

It is recommended, for the wind speed during installation activities, to have values different than the values obtained for the onset wind velocity of galloping (shown in Table 7).

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